2. Head loss due to change in velocity from V_4 to V_6.

From Curve "B" $H_{L6} - H_{L4} = (0.60 - 0.66) = \underline{0.06 \text{ ft.}}$

3. Head loss due to bends (90^o; 45^o bend)

From Curve "C"

-- using largest velocity between V_7 and V_6 for 45^o bend

$45^o \quad H_L = \underline{0.10 \text{ ft.}}$

-- using largest velocity between V_8 and V_6 for 90^o bend

$90^o \quad H_L = 0.10 \times 2.0 = \underline{0.20 \text{ ft.}}$

4. Head loss due to secondary flows

From Curve "D"

$$\left.\begin{array}{l} Q_7 = 1.86 \\ \\ Q_8 = 1.68 \end{array}\right] \text{ secondary flows}$$

$Q_y = 13.02$ upstream flow

$$\frac{Q_7}{Q_y} = \frac{1.86}{13.04} = 0.14 = 14\%$$

-- using V_4 for largest velocity

$H_L = \underline{0.08 \text{ ft.}}$

$$\frac{Q_8}{Q_y} = \frac{1.68}{13.02} = 0.13 = 13\%$$

H_L = <u>0.07 ft.</u>

Total H_L = <u>0.74 ft.</u>

<u>Friction Head Losses</u>:

Friction head losses are computed by the formula:

$$H_f = S_f \times L$$

where Hf = head loss in feet

L = pipe length in feet

$$S_f = \frac{Q \times N^2}{1.486\ (AR^{2/3})}$$

The friction head losses for each pipe section in the sample problem are shown below.

1 - 2 $S_f = \dfrac{7.53(0.013)^2}{1.486(0.566)} = 0.014$

$H_f = 0.014 \times 20.5' = 0.28$ ft.

2 - 3 $S_f = \dfrac{8.93(0.013)^2}{1.486(0.919)} = 0.007$

$H_f = 0.007 \times 178.1' = 1.25$ ft.

3 - 4 $S_f = \dfrac{10.64(0.013)^2}{1.486(0.919)} = 0.01$ ft.

$H_f = 0.01 \times 17' = 0.17$ ft.

5 - 4 $S_f = \dfrac{2.40(0.013)^2}{1.486(0.566)} = 0.001$

$H_f = 0.001 \times 17.5' = 0.02$ ft.

4 - 6 $S_f = \dfrac{13.04(0.013)^2}{1.486(1.979)} = 0.003$

$H_f =$ 0.003 x 92' = 0.31 ft.

7 - 6 $S_f = \dfrac{1.86(0.013)^2}{1.486(0.566)} = 0.001$

$H_f =$ 0.001 x 167' = 0.167 ft.

8 - 6 $S_f = \dfrac{1.68(0.013)2}{1.486(0.566)} = 0.001$

$H_f =$ 0.001 x 11' = 0.01 ft.

6 - 9 $S_f = \dfrac{16.58(0.013)^2}{1.486(3.588)} = 0.002$

$H_f =$ 0.002 x 31' = 0.06 ft.

Hydraulic Gradient Summary

The analysis of the hydraulic gradient computations is best performed with a summary sheet as shown in Figure 6-4. This permits easy comparison of hydraulic elevations with flowline elevations and the elevations of top of grates. The analysis is helped visually by plotting profile views (such as Figures 6-5, 6-6 and 6-7) showing the relative elevations of the various elements.

Gutter Flow Depths and Spreads

The curb, gutter and inlet hydraulics are analyzed to determine their capacities, flow depth and spread. Such evaluations are made below for inlets 2 and 7.

HYDRAULICS OF INLET 2 (45^o tilt-bar grate -- 1.7' x 3')

$Q = 1.4$ cfs

$d = .145'$, $T = 6.96$ ft., $S_x = 0.0208$,

$Z = 48$, $S_o = 0.013$

From Figure 6-26a:

$$\frac{W}{L'} = 0.31 \qquad W = 0.93$$

W_E = effective width = W + W = 1.7 + 0.93 = 2.63'

$$E_o = \text{Inlet efficiency} = 1 - \left(1 - \frac{W_E}{T}\right)^{8/3} = 1 - \left(1 - \frac{2.63}{6.96}\right)^{8/3} = 0.72$$

From Figure 6-28:

$K = 1.23$

$$V_F = \text{Frontal Velocity} = \frac{2KQZ}{T^2} = \frac{(2)(1.23)(1.4)(48)}{(6.96)^2} = 3.47 \text{ f/s}$$

From Figure 6-26b:

R = Reduction factor = 1.0

E = Hydraulic efficiency = E_o R = (0.72)(1) = 0.72

Q_i = Inlet capacity = EQ = (0.72)(1.4) = 1.01 cfs

Q_c = Carryover = Q - Q_i = (1.4 - 1.01) = 0.39 cfs

CK safety factor against clogging

$$\bar{d} = \left(T - \frac{W}{2}\right) S_x = \left(6.96 - \frac{1.7}{2}\right)(0.0208) = 0.127$$

$\bar{d}$ = (depth at mid point of grate)

$$V_F = \frac{QE_o}{w\bar{d}} = \frac{(1.4)(0.72)}{(1.7)(0.127)} = 4.67 \text{ f/s}$$

From Figure 6-26b:

1.7 feet of the grate is used at $V_F = 6.3$ f/s

$$S.F. = \frac{(6.3)}{(4.67)} = 1.3 \text{ against clogging}$$

HYDRAULICS OF INLET 7: (45^o tilt-bar grate -- 1.7' x 3')

$Q = 1.79$ cfs, $d = 0.165'$, $T = 7.92'$
$S_x = 0.0208$ f/f, $Z = 48$

From Figure 6-26a:

$\frac{W}{L_1} = 0.31$, $W = 0.93$

$W_E = 2.63$, $E_o = 1 - \left(1 - \frac{2.63}{7.92}\right)^{8/3} = 0.66$

From Figure 6-28:

$K = 1.24$

$$V_F = \frac{(2)(1.24)(1.79)(48)}{(7.92)^2} = 3.40 \text{ f/s}$$

From Figure 6-26b:

$R = 1.0$

$E = (0.66)(1) = 0.66$
$Q_i = EQ = (0.66)(1.79) = 1.18$
$Q_c = (1.79 - 1.18) = 0.61$ cfs
CK against clogging

$$\bar{d} = (7.92 - \frac{1.7}{2})(0.0208) = 0.147$$

$$V_F = \frac{(1.79)(0.06)}{(1.7)(0.147)} = 4.73 \text{ f/s}$$

$$S.F. = \frac{6.1}{4.73} = 1.33 \text{ against clogging}$$

Probable Depth at Sumps

Inlet 1 is in a pocket at the lower end of a swale. Inlets 3 and 8 are at the pavement sag. Inlet 5 is at the sag but in the intercepting ditch on the south side of the roadway. The probable depths of design flow at each sump are computed as follows:

INLET I (Std. PW-BD-1 grate -- 2.37' x 3.5')

Given: Q = 7.53 cfs, S = 0.013

Perimeter = 2B + 2L = (2)(2.37) + (3.5)(2) = 11.74'
1/2 perimeter = 5.87

$$Q_p = \frac{7.53}{5.87} = 1.28 \text{ cfs/f}$$

From Figure 6-16:

d = 0.5'

(This is satisfactory since grate is pocketed at end of swale.)

INLET 3 (Std. grate -- 1.7' x 3')

Given: S = 0.013

Q_i = (0.449)(3.85) = 1.73 cfs
Q_c (carryover from Inlet 2) = <u>0.39</u>
Q Total = 2.12 cfs

Perimeter = (2)(1.7) + 3.5 = 6.9 f
1/2 P for clogging = 3.45 f

$$Q_p = \frac{2.12}{3.45} = 0.61 \text{ cfs/f}$$

d = 0.37' = Depth of water above grate

INLET 5 (Std. PW-BD grate -- 2.37' x 3.5')

Given: $Q = 2.4$ cfs, $S = 0.013$

Perimeter = 11.74 x 1/2 P = 5.87

$$Q_P = \frac{2.4}{5.87} = 0.40$$

d = 0.30 (O.K. since at end of swale)

INLET 8 (Std. grate -- 1.7' x 1.3')

Given: $Q_8 = 1.68$, $S = 0.013$

Perimeter = 6.9 f.

1/2 P = 3.45 f.

$Q_c = 0.61$, $Q_T = 2.29$, $Q_P = .667$

d = 0.38 (O.K. since at end of swale)

Subsurface Drainage

The purpose of subsurface drainage is to remove detrimental groundwater in order to assure a stable roadbed and side slopes. Most often, those points or areas that require specific subsurface drainage will be determined during the process of field investigations. However, there are times when water-bearing formations are not revealed until construction has begun. "Bleeding" backslopes and apparent spring interceptions are two of the more obvious indications of this excessive groundwater.

Information on the potential need for underdrain treatment comes from several sources. The Materials Laboratory provides information on existing water tables and soils conditions. Maintenance personnel will recognize and identify unstable roadbed and surfacing conditions attributable to groundwater. And designers may locate potential trouble spots during field reviews.

Perforated pipe underdrain should be installed to intercept most subsurface drainage waters. These pipes, available in sizes of 6-inch, 8-inch and larger diameters, may form a network of interceptors and be routed to a central point of collection and outfall. Installation of underdrains should be as shown in the Standard Sheets.

Subsurface drainage is of primary importance in curb and gutter sections of the roadway. Where there is evidence of subsurface water, perforated underdrain pipe should be installed so as to empty into the drainage structure connecting the storm drain pipe -- at an elevation well above the flow line and as high as practicable above the storm drain pipe. This will eliminate the possibility of perforated pipe emptying against a head developed in the drainage structure and forcing the water out the perforations.

Surface drainage should not be permitted to discharge into an underdrain. Underdrains should be permitted to empty into a roadway drainage system only where the outfall is not against a head.

Normally, underdrains of 6-inch diameter should be allowed to run no more than 500 feet. For distances over 500 feet, the minimum diameter should be 8 inches. A minimum of 0.20 percent pipe gradient is recommended.

Cleanouts should be installed on long runs of underdrain pipe. These may consist of a vertical small-diameter steel water pipe through which a pressure hose can be connected to flush out the system. The vertical pipe should be capped. Outlets for underdrains normally should be spaced no greater than 1,000 feet.

Pipe Design

Previous sections discussed the determination of pipe sizes in relation to estimated design discharge. This section is directed to design considerations related to physical standards for pipes and to criteria for installation.

Types of Pipe

Pipes are manufactured in various sizes and shapes and from various types of materials. Types of pipe commonly used in Delaware include:

- Reinforced Concrete Pipe (Round)
- Reinforced Concrete Pipe (Horizontally Elongated)
- Corrugated Steel Pipe (Round)
- Corrugated Steel Pipe Arch
- Corrugated Aluminum Pipe (Round)
- Corrugated Aluminum Pipe Arch
- Corrugated Steel Pipe Downspouts
- Perforated Pipe Underdrains

Large-size corrugated metal structural plate pipes also are available, but usually they are a special design responsibility of the Bridge Section.

Pipe arches and horizontally elongated pipes normally are used only where there is a limited amount of headroom and minimum cover over the culvert.

For corrugated steel pipes, several optional special treatments may be called for:

- Bituminous Coated -- a complete coating of bituminous material over galvanized steel.

- Bituminous Coated with Paved Invert -- additional protection for the pipe invert, coated over galvanized steel.

- Full Paved -- paving the inside total circumference of galvanized steel pipe to the depth of corrugations.

- Aluminized -- a thin coating of aluminum over steel (instead of galvanizing) as protection against corrosion.

The bituminous and aluminum coatings provide greater resistance to corrosion than is found with plain galvanized pipe. They are desirable with conditions of corrosive soils. For extremely corrosive conditions, corrugated aluminum pipe should be considered. The soils report provides a measure of corrosiveness of the soils along with guidelines for the most appropriate treatment.

The paved invert provides added protection to the flowline from the erosive action of sand and debris during high velocity flow. Additionally, a fully paved pipe demonstrates better hydraulic characteristics with a lower value of Manning's roughness coefficient.

The flowline slope of the pipe may influence the type of pipe. Concrete pipes normally should not be used when the velocity is over 10 feet/second. Straight-line metal pipes may be used with higher velocities, but consideration should be given to the use of stepped pipes and downspouts.

The type of pipe to be used should be based on the desirable hydraulic characteristics, soil conditions under which it must function, and the installation and maintenance cost. In Delaware, where the slopes of the pipes in a storm drainage system are relatively flat and pipe depths are minimal,